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SAFE LOADS ON DOG-LEG PILES

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SOIL MECHANICS AND FOUNDATIONS
DIVISION

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SAFE LOADS ON DOG-LEG PILES

James D. Parsons,[1] M. ASCE and Stanley D. Wilson,[2] A.M. ASCE

INTRODUCTION

In the driving of steel encased concrete or pipe piles it has long been the general practice to base their acceptance, among other things, on whether or not any portion of the bottom of such piles may be seen after they have been driven. If the alignment of a pile, after it has been driven, is such that it is not possible, through the aid of a light lowered into the pile, to see any portion of the bottom of the pile it is abandoned and considered to have no presumptive bearing value irrespective of its driving characteristics. This is accepted practice even where such piles meet refusal on rock at depths which are consistent with known depths to rock established by borings. This requirement, on the face of it, is without question extremely rigid, particularly where the lengths and cost of such piles are great and it places piles of this type at a distinct disadvantage over piles of a type which cannot be inspected after driving. In fact, in the instance of wood or H-piles little or no thought is given to alignment of the pile below ground in spite of the fact that substantial loads must be carried by the latter to justify their use. Yet, even in the light of the fact that we have no way of knowing the condition of such piles below grade, we often place complete confidence in them to carry substantial loads.

In the interest of economy, no doubt many engineers have raised a question about the validity of accepting as reasonable the existing rules governing the acceptance of open piles but unfortunately have been unable to demonstrate the degree of curvature existing in piles whose bottoms could not be seen and therefore could not assign a realistic bearing value to the piles. To permit the salvaging of such piles required first the development of suitable equipment which could be lowered into the piles to determine the curvature of the piles at various points along their length, and then the development of an analytical approach which could be utilized to evaluate the influence of such curvature on the load carrying capacity of the piles. Experience has dictated that substantial loads can be carried by piles having some curvature and it is the purpose of this paper to present a logical method of evaluating their capacity and thus provide a means of justifying their use, such approach having been verified by load tests.

The need for the development of both a means of measuring the

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the inclination of piles exhibiting curvature and a method of analyzing their load carrying capacity recently arose on two large New York City Projects. These will be referred to as Cases A and B. In the first of these two instances composite piles ranging from 80 to 140 feet in length were being driven. Out of the first 93 piles driven for this project 13 were found to exhibit curvature and their bottoms could not be seen. Since the contract drawings called for the driving of approximately 800 composite piles it became evident at the outset that a considerable savings in both cost and time might be accomplished if the dog-leg piles could be salvaged, particularly in the event the early indications in respect to the ratio of dog-leg to straight piles continued, since the cost of one of these piles was in excess of $500.

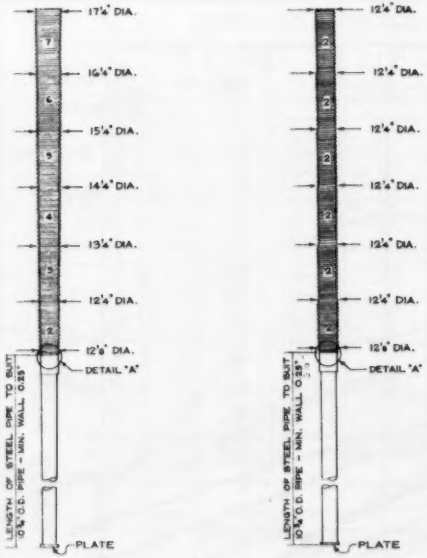
Description of Composite Piles—Case A

The lower portion of the composite piles analyzed consisted of closed-ended 10-3/4" O.D. steel pipe (3/8" wall thickness) and the upper portion of corrugated shell. Four thousand pound concrete was used in each and the piles were driven closed-ended as point bearing piles to carry a design load of 57½ tons each. Details of the piles, including the connection between the corrugated and pipe sections are shown in Figure 1. The original design for the piles called for a tapered shell in the upper portion but due to the extremely hard driving evidenced in the first piles driven this was later changed to a straight 12½" diameter shell in the subsequent piles driven. Even after this change was made it was found necessary to first drive an outer 14" diameter shell to a limited depth to eliminate friction along the upper portion of the pile, this shell being pulled in each case immediately after completing the driving of the pile, thus permitting the granular soil surrounding the shell to fill the annular space between the shell and pile. A total of six load tests, three on each of the two types of piles referred to, were performed in accordance with the requirements stipulated in the New York City Building Code. The maximum design load permitted for these piles was governed by the maximum allowable unit stress permitted by the Code for 4000 lb. concrete of f_c = 1000 lbs. per sq. in., the cross-sectional area of the lower 12-1/8" diameter shell of the upper section controlling.

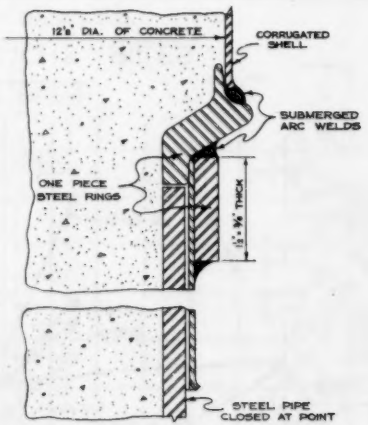
The load settlement diagram for each of the six load tests performed on straight piles are included in Figure 2. The loads were in each case applied in the increments stipulated by the Code to a maximum test load of twice the design load or 115 tons and as may be seen in Figure 2, the piles satisfactorily carried this load since the maximum deflection in each case was less than the maximum gross settlement of 1" permitted by the Code.

Soil Formation—Case A

The specifications governing the driving of the composite piles stipulated that the piles were to be driven through the fill and underlying sand and silt formation to rock or the gravel boulder formation directly overlying rock to a minimum penetration resistance of 8 blows per inch



DETAILS OF COMPOSITE TYPE PILES
UTILIZED IN BUILDING No 10



DETAIL "A"

CASE A PILES

FIGURE 1

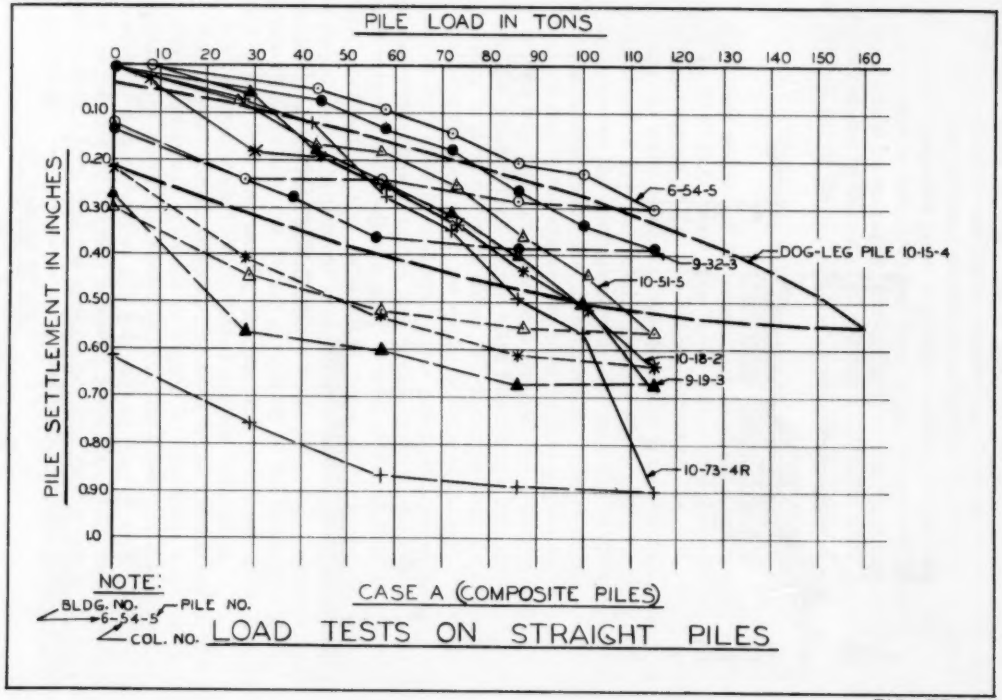
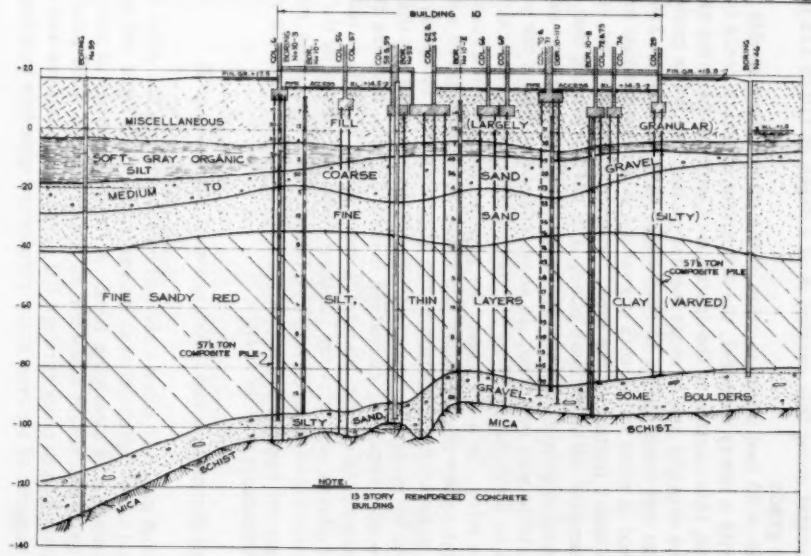

PILE LOAD IN TONS
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160
PILE SETTLEMENT IN INCHES
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.0
6-54-5
DOG-LEG PILE 10-15-4
9-32-3
10-51-5
10-18-2
9-19-3
10-73-4R
NOTE:
BLDG. NO.
PILE NO.
6-54-5
COL. NO.
CASE A (COMPOSITE PILES)
LOAD TESTS ON STRAIGHT PILES

FIGURE 2

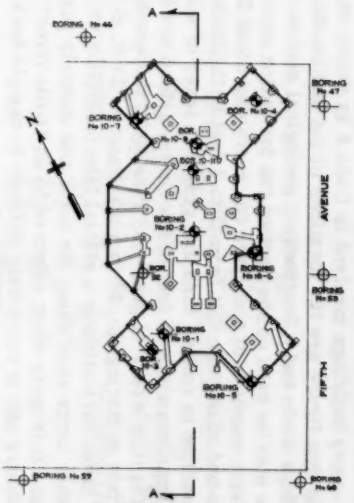
BORING LOCATION PLAN AND GEOLOGICAL SECTION



SECTION A-A

SCALE

CASE A



BUILDING No 10
FOUNDATION & BORING LOCATION PLAN

SCALE

FIGURE 3

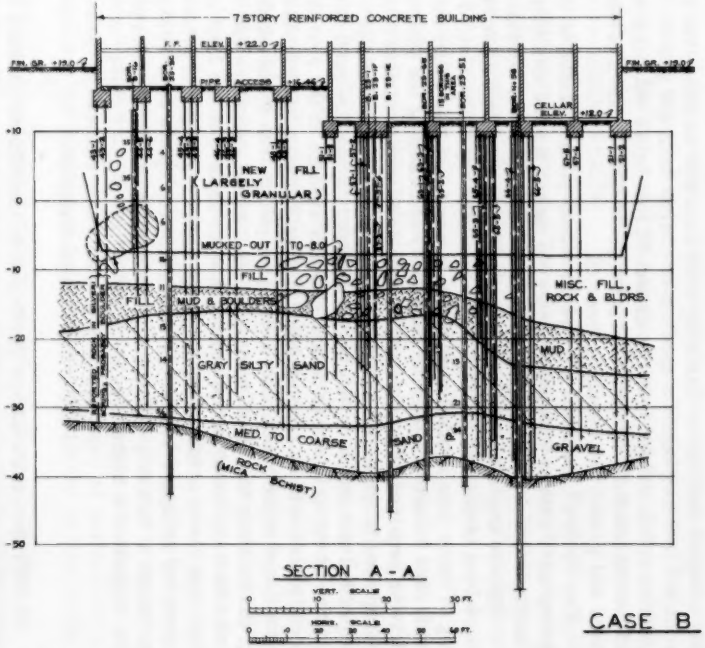
with a 15,000 ft. lb. hammer. A typical foundation plan of one of the three 13 and 14-story buildings involved in Case A is shown on Figure 3. Also shown in this figure are the locations of the undisturbed and exploratory borings made and a longitudinal geological section through the building. As may be seen on the section the formation prevailing in sequence with depth consists of a surface fill averaging about 20 feet in thickness, a thin bed of organic silt from 3 to 8 feet in thickness, about a 10 ft. bed of medium to coarse sand and gravel, a layer of silty fine sand averaging approximately 15 feet in thickness, a deep bed of sandy red silt containing thin layers of gray clay ranging in thickness from a minimum of 40 feet at Boring No. 46 to a maximum of 53 ft. at Boring No. 10-3. This latter formation is a glacial lake formation which extends over a rather large area along the north and easterly portions of Manhattan Island. It is quite often referred to as "bulls liver" by the construction industry and is extremely difficult to sample because of its sensitivity to vibration. Below the varved silt formation and overlying bedrock was encountered a bed of silty sand and gravel containing some boulders, the formation averaging about 8 feet in thickness. Bedrock of mica schist along this section is at a depth of 110 to 115 feet below existing grade.

Since the 40 to 60 foot upper section of the piles driven at this site were driven with a heavy mandrel, the alignment of which could be carefully controlled, this section in the instance of each of the piles analyzed was found to be straight and in most instances vertical. The curved or dog-leg section was found to be, in each case, in the lower 10-3/4" O.D. pipe section and in most cases this curvature was very gradual and limited to the lower portion of the pipe. When you consider the very high length diameter ratio of these piles, it is not surprising that as many were found to have some curvature, particularly in those instances where hard driving was required. In some cases the tips of the piles were found to be as much as $5\frac{1}{2}$ to 6 ft. out of line with a vertical through the tops of the piles. In a few cases the pipe section was not found to be tangent with the upper shell section and these were considered the most critical, as a substantial reduction in the allowable load to be carried by these had to be made. It is perhaps significant to point out that of the 63 piles at the site requiring curvature measurements, because the tips could not be seen with a light, about 45% were found to be less than 2 percent out of plumb, which is the limiting value permitted by the Code before compensating measures must be taken. Yet these piles would have been replaced had not the measurements been made, since no portion of their bottoms could be seen.

Soil Formation and Description of Piles--Case B

In the second group of piles analyzed by the writers the piles were closed-ended 10-3/4" pipe piles driven to rock or the sand and gravel formation immediately overlying rock. A limited number of such piles were driven as replacement piles for 25 ton creosoted wood piles which had been damaged in driving through fill containing boulders and broken rock for the support of a seven story reinforced concrete building. Cost

BORING LOCATION PLAN AND GEOLOGICAL SECTION

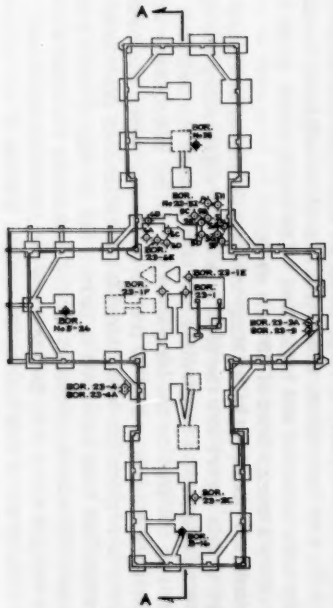


SECTION A - A

VERT. SCALE 10 0 30 FT.

HORIZ. SCALE 0 20 30 40 50 60 FT.

CASE B



7 ST. REINF. CONC. BLDG.
FOUNDATION & BORING LOCATION PLAN

SCALE 0 10 20 30 40 50 60 FT.

FIGURE 4

estimates dictated that in the event such replacement piles could be successfully driven through the areas where rock and boulders were encountered, a considerable savings could be accomplished over the removal of such by excavation, since portions of the adjacent foundation had already been installed and backfilled. The replacement piles were driven to carry a design load of 30 tons each.

The foundation plan for the building involved is shown in Figure 4. Also shown in this figure are the locations of the exploratory borings in the building area and a longitudinal section through the building. As may be seen on the section the formation prevailing in sequence with depth consists of a surface fill, 20 feet in thickness, which had been recently placed to replace the objectionable miscellaneous fill containing boulders which had been removed to permit the driving of wood piles. Below the new fill was encountered old fill containing rock and boulders. This bed averages about 8 ft. in thickness and, as above discussed, the failure to remove all of this bed in the central portion of the building where concentrations of rock were found was responsible for the wood pile breakage. Below the old fill is a bed of soft gray organic silt averaging approximately 4 to 8 feet in thickness. Some of the rock from the overlying fill was found to have penetrated into this formation. Under the organic silt bed was encountered about a 15 ft. bed of reasonably compact medium to fine silty gray sand and between this and bedrock a 2 to 8 foot layer of medium to coarse sand and gravel. Bedrock of mica schist ranged between 45 and 55 feet below cut-off level. Most of the successfully driven wood piles penetrated to the medium to coarse sand and gravel bed overlying bedrock.

The 10-3/4" diameter pipe replacement piles included in this second group were driven with points and even though a spud was employed to facilitate driving through the rock-boulder formation, considerable difficulty was experienced and a large percentage of the replacements were bent in penetrating this formation. During the course of the driving of these pipe replacement piles, it was found that by filling the piles with sand before driving, the possibility of crimping the pipe in the hard driving required in penetrating the boulders was minimized, this sand being readily blown out prior to filling with concrete. In spite of this expedient and others which were employed, it became evident during the course of the driving that a considerable savings in the cost of replacement piles could be accomplished if piles having some curvature could be utilized since practically every pipe pile driven in the central area of the building was found to have at least some curvature and the bottoms of many could not be seen. This we believe is the first project in this city where actual measurements fo the curvature in piles were taken and analyzed and for which permission by the New York City Department of Housing & Buildings was granted to utilize such piles for the support of a structure.

Deflection Measurements

In 1951 the junior author developed an instrument called a slope indicator which could be used under water and which could measure directly

the slope or inclination of steel sheet piling at various depths.[3] This inclination, however, could be measured only in a plane normal to the surface against which the instrument was in contact, and the range of tilt was limited to about eight degrees from the vertical. The tips of some of the piles described in this paper were found to be inclined at angles in excess of 15 degrees. It was, therefore, necessary to design and construct a new instrument which could measure the maximum inclination at any depth regardless of the orientation of the instrument with respect to the pipe. Fortunately it was necessary to ascertain the magnitude only and not the direction of this inclination.

The slope indicating system used in the revised instrument is basically the same as that devised for the earlier model and is based on the well-known Wheatstone Bridge circuit as illustrated in Figure 5a. For any given position of the pendulum arm in the slope indicating unit, there is only one position of the potentiometer arm such that no current flows through the neutral wire connecting the two arms. A sensitive galvanometer indicates when a balance point is attained.

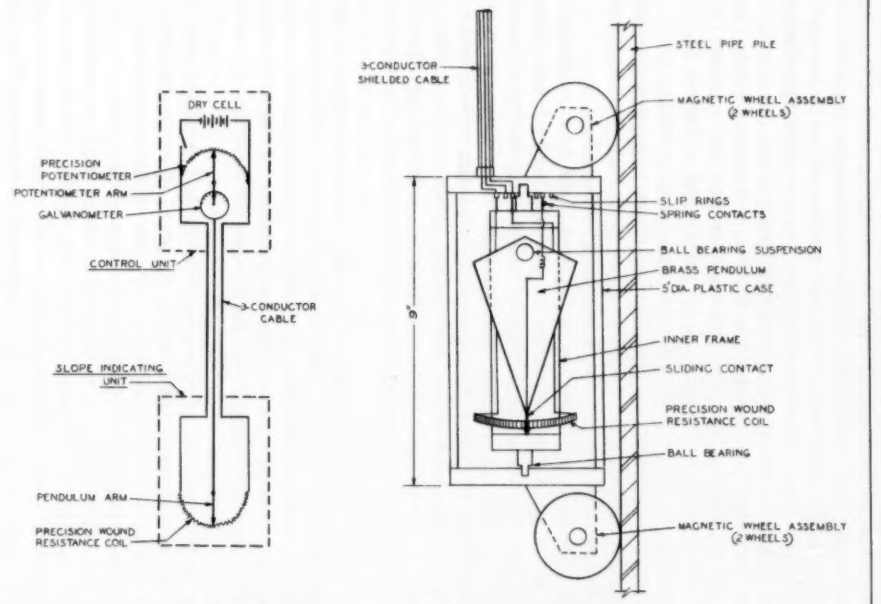
The slope indicator itself, shown in Figures 5 and 6, is contained inside a five-inch diameter plastic tube, nine inches long. A heavy brass pendulum with ball-bearing suspension is mounted on an inner frame which is free to rotate on ball bearings about its longitudinal or vertical axis. The tip of the pendulum slides along a precision wound resistance coil which is also attached to the inner frame. The electrical leads from the ends of the coil and from the tip of the pendulum are brought to the case through slip rings and thence to the surface of the ground in a 3-conductor shielded cable.

The watertight plastic case which encloses the inner frame is mounted on two sets of steel wheels. The wheel axles are made of Almico bar magnets. These magnetized wheel assemblies maintain a firm contact between the rim of the wheels and the inner surface of the steel pipe pile yet at the same time permit the instrument to be rolled up or down the entire length of the pile.

In operation the instrument is placed in a vertical position with the wheels in contact with the inside wall of the pipe. The non-magnetic pendulum assumes a vertical position under the influence of gravity, whereas the longitudinal axis of the inner frame remains parallel to the axis of the pipe. If the plane of maximum inclination is perpendicular to the surface against which the wheels are in contact, the horizontal axis of the pendulum will remain parallel to the wheel axles and there is no rotation of the inner frame. If the plane of maximum inclination shifts (as in a cork-screw pile), or if the instrument tends to rotate as it is being lowered, the inner frame rotates under the force of gravity until the pendulum swings in the plane of maximum inclination. Check observations were made by lowering the instrument down on several sides of some piles with no significant change in slopes indicated.

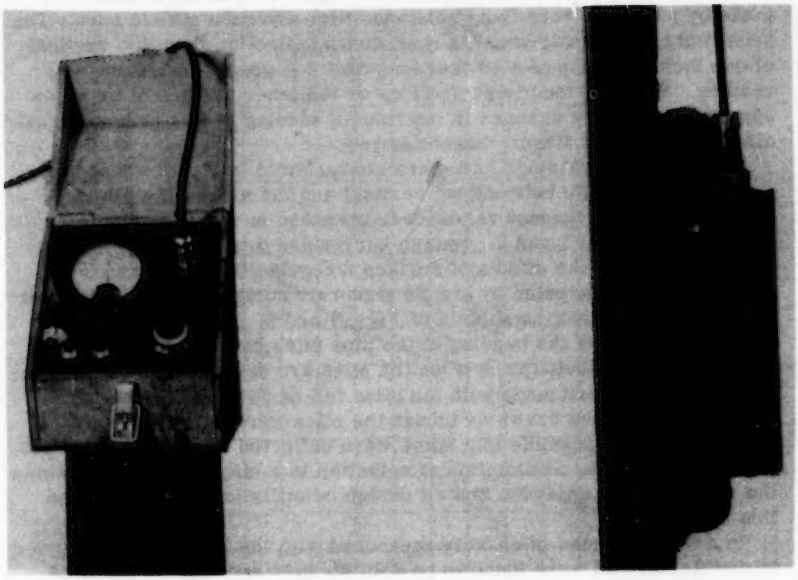
The downward descent of the instrument is stopped at desired intervals of depth and the instrument held stationary until the motion of the

3. S. D. Wilson, "Analysis of Existing Bulkheads Based on Deflection Measurements", paper presented at Boston Society of Civil Engineers, May 13, 1953.



DIAGRAMMATIC SKETCH OF SLOPE INDICATOR

FIGURE 5



a b

FIGURE 6

Photographs of slope indicator showing:

a. Control box which contains precision potentiometer and batteries.

b. Instrument containing non-magnetic pendulum which is lowered into pile.

pendulum has ceased. The precision potentiometer in the control box, which remains above ground at all times, is adjusted until the bridge circuit is in balance, and the corresponding dial reading is recorded. The true inclination of the pile at this depth is then obtained from the calibration curve of the instrument. No temperature or other corrections are required. The power supply is furnished by small dry cells enclosed in the control box, the power consumption being negligible.

The resistance coil across which the tip of the pendulum slides is made by winding a very fine resistance wire around a plastic core. The sensitivity of the instrument is such that a deflection from the vertical of one inch at the tip of a 40-foot long pile will produce a measurable reading. Since surface imperfections on the inner surface of the piles sometimes produce changes in instrument reading of this order of magnitude, greater sensitivity is not desired.

After the dial readings have been converted to inclination tangents (tangent of the angle between the vertical and the axis of the pile), a plot of inclination tangent vs. depth is prepared as shown on the left hand plots of Figs. 7 and 8. Smooth curves are drawn through these points to iron out the effects of surface irregularities after which the deflections are computed by simple algebraic computations, and plotted as shown on the right hand plots of Figs. 7 and 8.

The direction of the bending in the pipe piles cannot be ascertained except by visual observation when the piles are dewatered. Most of the measurements were made with the piles full of water. The deflection diagrams have been drawn as though the piles were deflected in one plane only. It is possible that some were deflected in a cork-screw pattern. However, the assumption of selection in a single plane represents the most critical condition and all design computations were made on this assumption.

In a few instances piles were measured with the slope indicator prior to dewatering and were found to be both straight and plumb. Some of these piles were later dewatered, thus affording a positive check on the reliability of the instrument.

The slope indicator is light and even in the case of a 150 ft. pile can be easily handled by one man. An additional man, of course, is required to record the readings. It was found in the case of the piles described earlier under Case A that as many as 30 piles could be measured in an eight hour period taking readings at intervals of 5 ft. in depth.

Lateral Restraint Provided by Soil

The ability of curved piles to carry satifactorily significant loads even though badly inclined or dog-legged is due to the fact that the lateral restraint to movement provided by the soils surrounding the piles is substantial and usually considerably in excess of that required to provide stability. This is a factor not generally realized or taken advantage of by the engineering profession. Reference can be made to numerous successful installations of very heavily loaded long piles driven through deep deposits of extremely soft organic silt as evidence of

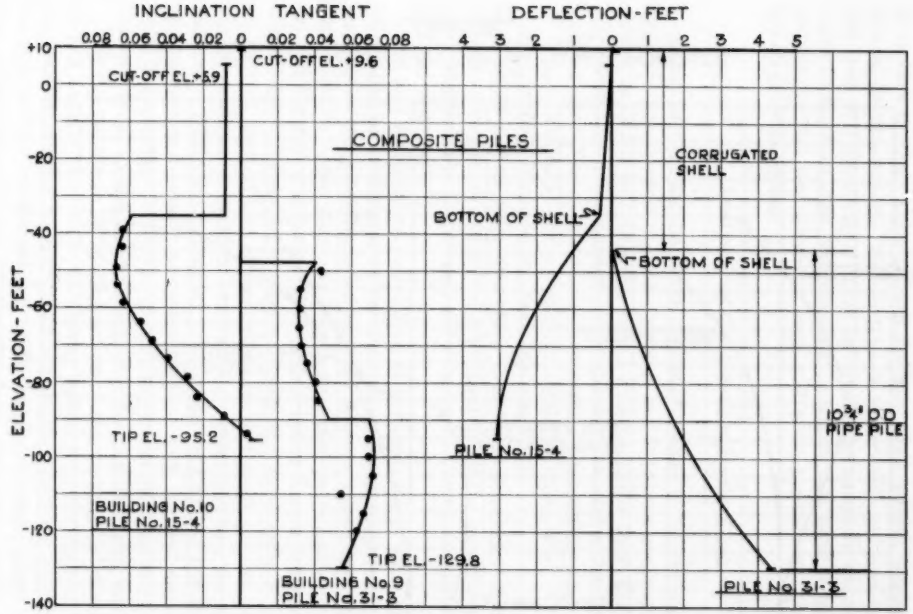


FIG. 7 - TYPICAL DEFLECTION MEASUREMENTS
CASE A

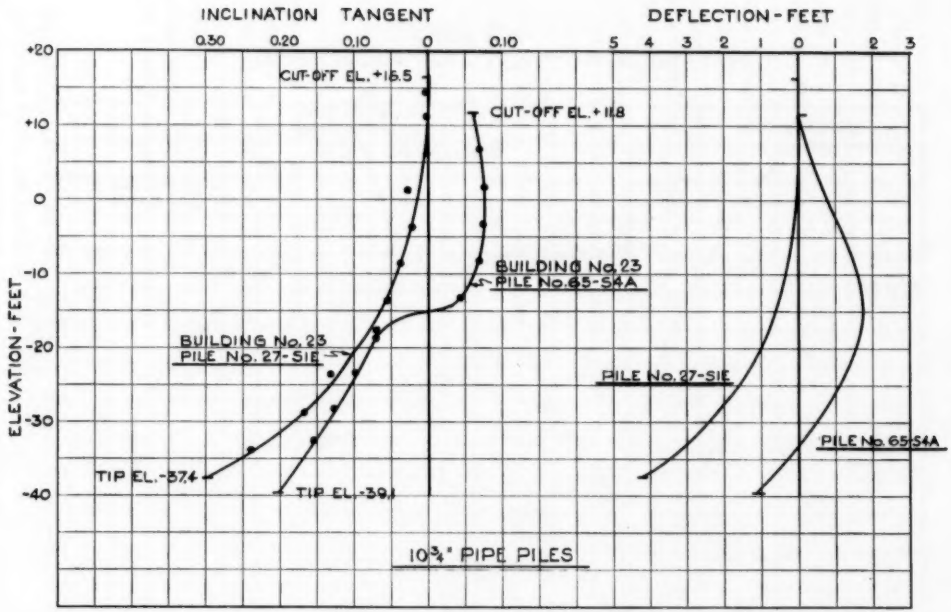


FIG. 8 - TYPICAL DEFLECTION MEASUREMENTS
CASE B

the presence of this restraining force.[4] The recent description in "Civil Engineering"[5] of a heavy paper-making machine in Oslo, Norway supported on 1-1/4" diameter steel rods pushed and driven through approximately 150 feet of soft clay and loaded to about 8 tons each is also an unusual example of this phenomenon. Undoubtedly many of these rods must have some curvature since it is highly improbable that they could be driven through such a great depth of clay without experiencing some lateral displacement. Yet they have reportedly carried the heavy loads involved satisfactorily. It is believed that as more cases of this type come to the fore, the profession as a whole will have less hesitancy to accept the fact that even the very soft soils provide substantial restraint to lateral movement.

Analyses of Case A Composite Files

In the determination of the allowable loads to be carried by the dog-leg piles considered in both Cases A and B it was necessary to demonstrate, in order to fulfill the requirements of the New York City Building Code, that the unit stresses in the piles were within the limits permitted by the Code. The allowable unit stresses controlling are:

1) f_c = 1000 lbs. per sq. inch for 4000 lb. concrete
2) f_s = 9000 lbs. per sq. inch for steel.

An acceptable analysis, therefore, had to be developed to demonstrate that the stresses in the piles, irrespective of the results of load tests, were within these limits and the results had to be transmitted to the City for approval.

The soil formations prevailing at the location of each of the sites involved had to be defined by an adequate number of borings. The upper portions of the piles included in Case A were surrounded by sand while their lower or pipe sections were embedded principally in varved red silt and the physical properties of this latter formation had to be obtained. Accordingly a 3½" diameter undisturbed sample boring located in the central area of one of the buildings was made for the purpose of obtaining continuous undisturbed samples from this bed for laboratory analyses. While the principal properties involved in the analyses were the strength and elastic properties of the silt, natural water content, liquid and plastic limit determinations, consolidation and triaxial compression tests of the quick and consolidated quick type were performed.

The curvature in the pipe section of the composite piles, while gradual and usually limited to the lower end of this section, in some instances was found to extend to the joint between the shell and pipe sections and the pipe section was found not to be tangent with the shell section. The piles requiring analyses were, therefore, divided into two groups, namely:

1) Those in which the upper portion of the pipe section was found to be tangent to or on the axis of the shell section, and

4. "Influence of Soft Ground on the Design of Long Piles" by George W. Glick. Proceedings of 2nd Intern. Conference on Soil Mechanics & Foundations, Vol. IV.
5. "Civil Engineering", April 1953 p. 60 "Machine Foundation Supported by Steel Rods Driven Through Clay", by Ernest E. Howard.

2) Those exhibiting a definite break in the alignment between the pipe and shell sections in which case the upper portion of the pipe section was found not to be tangent to the axis of the shell section.

Fortunately, the latter group was substantially in the minority since an appreciable reduction in the allowable load to be carried by these piles had to be made. A description of the analyses for each of the above two cases referred to is included in subsequent sections.

a) Basic Assumptions. In Figure 9 are shown the forces acting on the upper and lower sections of a typical Case A dog-leg pile of the type above described under (1). The pile in its original position before being loaded is indicated by the solid lines. Under the influence of an applied vertical load at the top, the shell section being vertical deflects axially. The lower inclined section, however, since its bottom is sufficiently embedded in firm material to prevent lateral movement at this point and since the deformation of its top is almost entirely vertical, will experience its maximum lateral movement at its center as shown by the dashed line in Figure 9. In so doing, it will develop pressures in the soil proportional to its displacement which will be a maximum at its center, zero at its tip and essentially vertical at the top and the resulting pressure distribution in the soil can thus be approximated by a sinusoidal curve such as is shown on the pipe section in Figure 9. Since the shell section moves only vertically additional assumptions had to be made to determine the soil pressure along this section and these as well as others which were required to permit a solution to be developed were:

A) The joint between the pipe and shell sections of the pile is considered to be hinged.

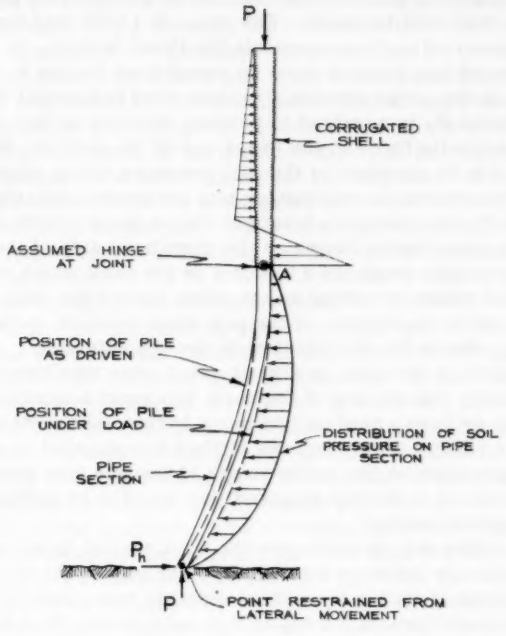
B) Due to the curvature in the pipe section lateral restraint from the soil along the lower portion of the straight shell section is required to produce stability. The available pressure providing this lateral restraint was assumed as $1/2\ \gamma h$.

C) The distribution of restraining force at the bottom of the shell section was assumed to approach as nearly a concentrated reaction as permitted by the maximum available soil pressure assumed in (B).

D) No horizontal force was assumed to be available at the top of the the pile from the pile cap or strut and the only force considered at this location was the vertical pile load.

In assuming that the connection between the shell and pipe sections is hinged recognition of some weakness of the pile at this point is taken into account. Such an assumption, in effect, renders necessary the mobilization of greater stresses in the soil along the lower portion of the shell section than will actually be mobilized and thus the bending moment in this section is increased above actual and the allowable load on the pile reduced. The true stresses at this point are not known and for this reason a conservative assumption had to be made and in making this assumption the problem was reduced to one which is statically indeterminate to the first degree.

In connection with the distribution of soil pressure on the pipe the influence of assuming either a uniform, triangular, or trapezoidal



FORCES ACTING ON CASE A PILE

(PIPE SECTION TANGENT TO SHELL SECTION)

FIGURE 9

distribution was also investigated and found to have a negligible effect on the results.

In the instance of above assumptions (B) and (C) it should be pointed out that the maximum available restraining force from the soil on the upper or shell section is the difference between the full passive and active pressures, or approximately $3\gamma h$ since the soil surrounding this section is sand. A value of $1/2\gamma h$ was taken in the analyses. This was done because of uncertainties regarding the true stress conditions prevailing at the joint and the fact that the lateral deformation in the lower portion of the shell will be small. The value of $1/2\gamma h$ was considered sufficiently conservative to compensate for these factors. In regard to the distribution of this force it must be recognized that as a result of the curvature in the lower section a concentrated horizontal force at the top of this section P_r is required to produce stability in this section. An equal and opposite force at the lower end of the shell is, therefore, required and this is provided by the soil pressure acting against the shell. Since the maximum restraining soil pressure available was assumed as $1/2\ \gamma h$, the pressure must act over a finite length of the pile. The triangular distribution shown on the right hand side of the shell results. This pressure produces a moment in the shell which must be resisted by soil pressure acting on the other side of the shell the moment of which about the bottom of the pile must be equal to the moment of the former. Hence the distribution on the opposite side.

Since the shell of the piles in almost every case was found to be vertical it is unlikely that the top of the shell will exert a horizontal force on the pile cap or have a tendency to move horizontally under a vertical load. This possibility, however, was further investigated by assuming that the soil pressure on the entire length of the pile was sinusoidal, and the horizontal force at the top required was found to be sufficiently small to permit neglecting.

The pipe section was in each case the first section to be analyzed. Since it is statically indeterminate to the first degree the horizontal force P_r required at its top to produce stability was taken as the redundant. The vertical force at the top of this section was first taken as the full design load of 115 kips and later reduced if the resulting stresses in the section were found to be in excess of the allowable. The method of analyses consists of determining the deflection of point "A" at the top of the pipe when force P_r equals zero, then calculating the value of P_r required to eliminate this deflection. In the analysis for P_r the effects of axial and shearing stresses in addition to bending stresses were considered. It was observed, however, that by omitting these, the computed value of P_r would be influenced only by about 5%. Consequently, in the analyses of the other piles these terms were omitted.

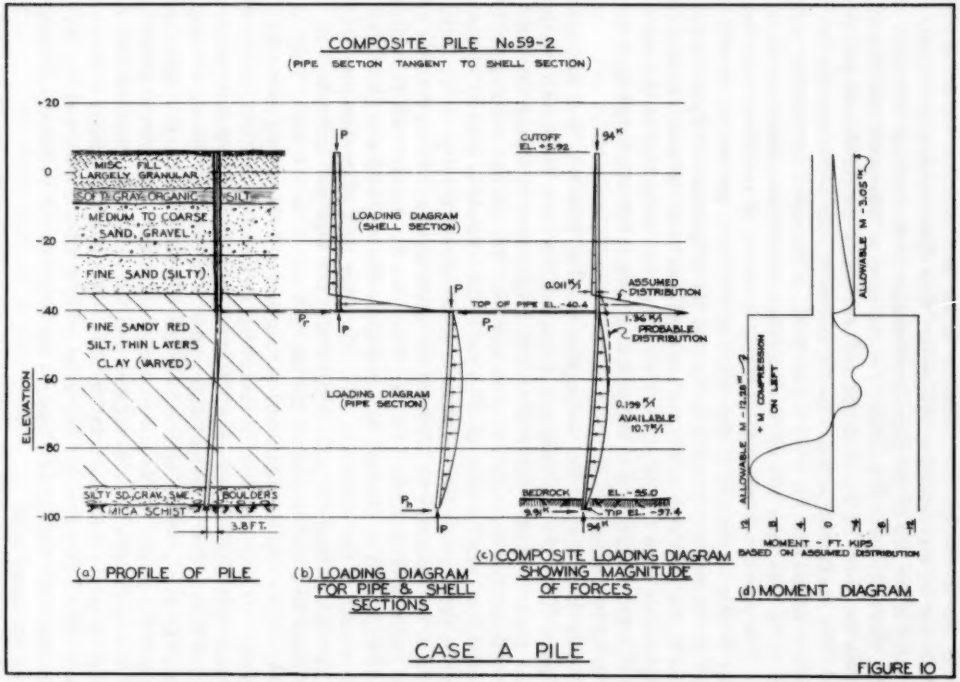
b) Pipe Section Tangent to Shell Section. In Figure 10a is shown a sketch of inclined composite pile No. 59-2 which falls into Category (1) above discussed, since as may be seen the pipe section is tangent to the shell section. In Figure 10b are shown the force diagrams for both the shell and pipe sections of this pile. In the treatment of the pipe section, it was found in first considering the influence of the full design load of 115 kips the value of the redundant, P_r was found to be 3.10 kips. The

soil pressure exerted on the pipe under this load is 251 lbs. per sq. ft. and this produces a combined stress from bending and axial load of 1200 psi in the concrete. Since this stress exceeds the allowable, the load was reduced from 115 kips to 96 kips to satisfy the stress limitations in this section. It was later found necessary to further reduce this load since it was found that the stresses in the shell section were controlling.

It should be mentioned at this point, that this analysis is strictly correct only if the top of the pipe is prevented from moving laterally. Actually, there is a small amount of movement at this point and this decreases the value of P_r obtained and increases the maximum soil pressure. The maximum soil pressure, however, can never exceed the pressure calculated for $P_r = 0$ and in all of the piles analyzed it was found to be less than 10% of the available restraining pressure from the soil. The average overburden pressure, alone, at the location of the center of this section is of the order of 3000 lbs. per sq. ft. It will be shown in the section which follows that the magnitude of P_r controls the stress in the concrete in the corrugated shell section and, consequently, the allowable load on the pile. Therefore, any reduction in the value of P_r will reduce stresses in the pile and permit the allowable load on the pile to be increased.

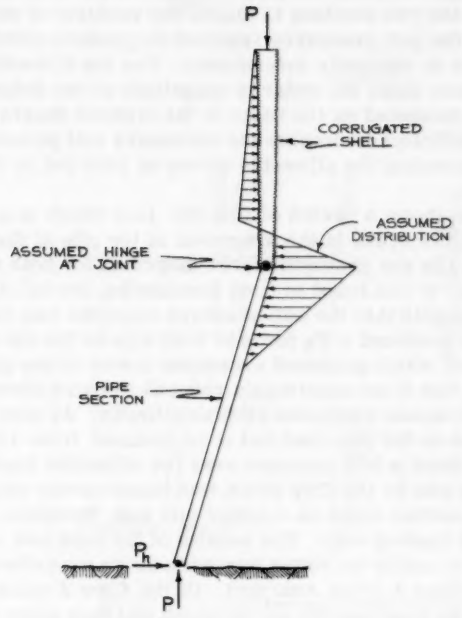
The loads acting on the corrugated shell of Pile No. 59-2 are shown in Figure 10b. The distribution of loading has been discussed earlier in this paper and the determination of magnitudes is straightforward as the shell section is statically determinate. The stress resulting from the axial load and bending is 1015 psi if the allowable load of 96 tons based on the pipe section results is used. Consequently, to limit the stress in the concrete to 1000 psi the permissible load on the pile had to be reduced from 96 to 94 kips. This was the design load recommended and approved by the city and in Figure 10c and 10d are shown the final results of the analyses. The allowable load of 94 kips requires a soil pressure along the pipe section of 0.199 kips per foot which is equivalent to a unit stress of 222 lbs. per sq. ft. in the soil. This is obviously only a small fraction of the amount of restraining pressure available. It will be noted in reviewing Figure 10c that the estimated probable distribution has been indicated by a dashed line and the assumed by a full line. The principal difference is in the distribution of loading around the joint. While this difference was fully realized in developing the analysis given, the assumed distribution was required to permit a practical approach to be developed and the results so obtained are recognized to be on the conservative side. The moment diagram given in Figure 10d is based on the assumed loading distribution and the stress in the concrete was determined on the basis of the combined bending stress and axial load.

c) Pipe Section not tangent to axis of Shell Section. In Figure 11 are shown the forces acting on a typical Case A dog-leg pile exhibiting a definite break at the joint in the alignment of the pipe and shell sections. Under the influence of a vertical load at the top, the shell section being vertical deflects axially. The straight lower inclined section, however, under the influence of a vertical load at its top will deflect laterally. The pipe section alone was analyzed under the forces shown in Figure 11



CASE A PILE

FIGURE 10



FORCES ACTING ON CASE A PILE

(PIPE SECTION NOT TANGENT TO SHELL SECTION)

FIGURE 11

and was found to have relatively large bending deflections under this distribution of forces. On this basis it was evident that the pipe was less rigid than the surrounding soil and, therefore, the soil reaction on the pipe will be concentrated near the upper end. Therefore, the distribution was assumed to be triangular and limited to the upper 1/3 of this section, as shown in Figure 11.

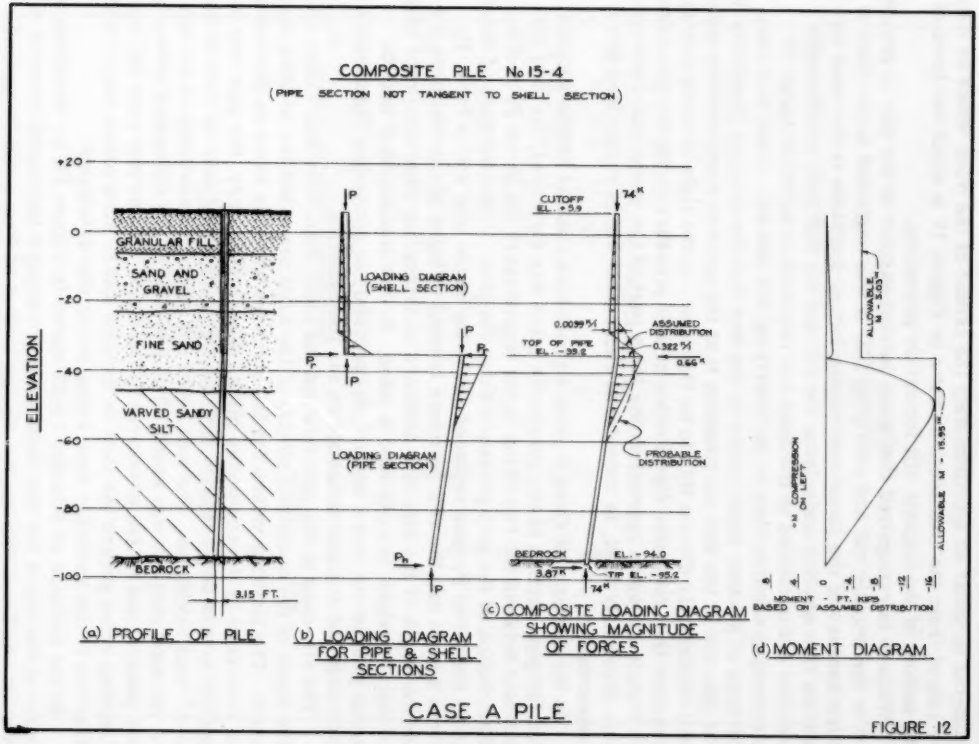
The distribution of soil pressure along the shell section is the same as that above discussed for Pile No. 59-2. On the basis of this distribution, that above described for the pipe section and the assumption that the joint between the two sections is hinged the problem of determining the magnitude of the soil pressures required to produce stability reduces itself to one which is statically determinate. For the allowable loads considered for these piles the order of magnitude of the deformations at the joint were computed on the basis of the moment diagram resulting and found to be sufficient to develop the necessary soil pressure required without exceeding the allowable stress of 1000 psi in the concrete in the shell.

In Figure 12 is shown a sketch of Pile No. 15-4 which is a specific case where a definite break in the alignment of the pile at the joint was found. In Figure 12b are shown the force diagrams for both the shell and pipe sections. It was found in first considering the full design load of 115 kips on this pile that the soil pressure required was 500 lbs. per linear foot. This produced a P_h force of 6.02 kips at the tip which resulted in a moment which produced excessive stress in the pipe. The loads, therefore, had to be accordingly reduced to those shown in Figure 12c to keep the stresses within the allowable limits. As may be seen the allowable load on the pile load had to be reduced from 115 to 74 kips. This represents about a 25% increase over the allowable load of 60 kips approved for this pile by the City which was based on our original premise that the pipe section acted as a batter pile and, therefore, could be subjected to axial loading only. The results of the load test later discussed completely justify the latter approach above described.

d) Number of Case A Piles Analyzed. Of the Case A piles measured, 29 were found to be less than 2% out of plumb and thus were approved without analyses, for a full design load of 57½ tons, 16 were assigned loads between 75% and 100% of the full design load and 16 were approved for 50% to 75% of the design load. Only two of the piles measured had to be rejected.

Analyses of Case B Files

As described under "Soil Formation and Description of Piles—Case B" the second group of piles analyzed by the writers were 10-3/4" diameter steel pipe piles which were required to be driven through fill and organic silt containing a sufficient quantity of boulders to produce at least some curvature in almost every one of the piles driven. As they were driven as replacements for wood piles which had been broken in penetrating these formations they each were only required to carry a design load of 30 tons. A typical example of the curvature resulting from the hard driving required is exhibited in Pile No. 15-S2 shown in



CASE A PILE

FIGURE 12

Figure 13. Also shown in this figure are the forces acting on the pile.

While the curvatures in the piles analyzed at this site in some cases were excessive in most instances they were found to be uniform and to extend throughout the entire length of the pile. The maximum lateral deflection in the piles under the influence of load could, therefore, be expected to occur at approximately the center of the piles which as may be judged from the soil section shown in Figure 13 is about the level of the bottom of the organic silt formation prevailing.

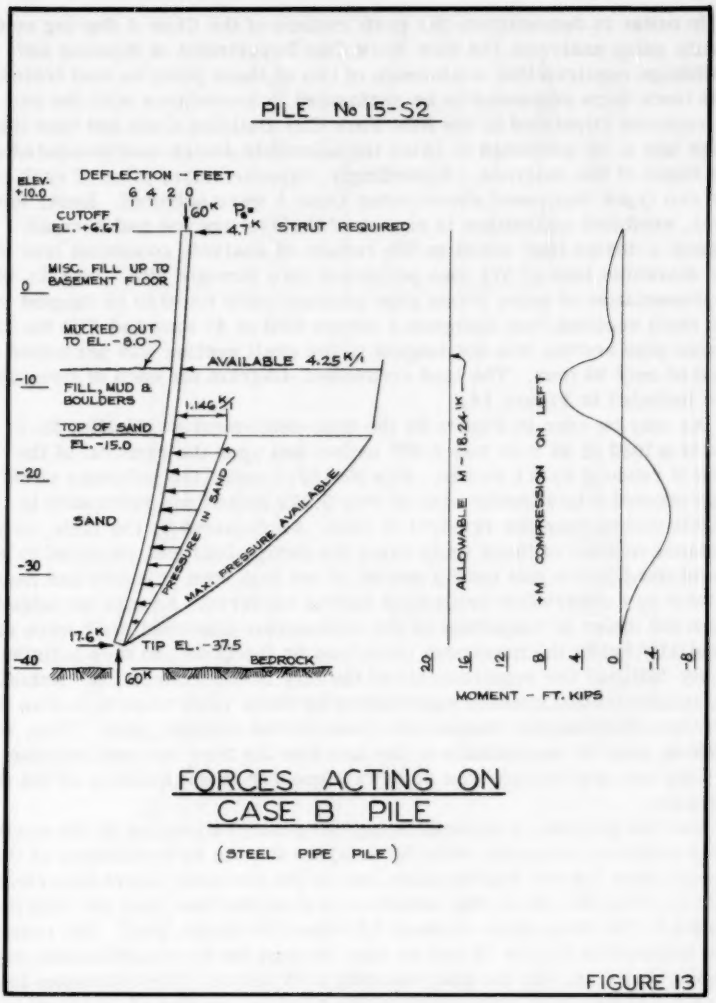
While it is recognized that some lateral support to the pile is provided by the organic silt and overlying fill, it was neglected in the Case B piles because of the large restraining forces available in the sand formation immediately underlying the silt and the fact that considerably smaller deformations in the sand are required to mobilize large restraining pressures than in the overlying silt and fill. Also, and even perhaps of greater significance is the fact that the relative flexibility of the pile and of the soil surrounding it at the point of maximum curvature will necessarily dictate this to be the case. In the light of these considerations the triangular distribution of soil pressure along the pile shown in Figure 13, as an approximation for a part of the sinusoidal distribution previously used, is considered reasonable but is recognized to be conservative.

In the analysis of Case B piles a procedure similar to that adopted for the pipe section of the composite piles was employed, the only difference between the two being in the treatment of the force P_r and the distribution of the soil pressure along the pile. In the instance of these piles the force P_r was replaced at the top of the pile with a force P_s from a strut and the soil pressure instead of being sinusoidal over the entire length of the pipe was assumed as triangular over the lower half of the pile which is embedded in sand. In this connection it may be again pointed out that a uniform trapezoidal, or sinusoidal distribution will have but a minor influence on the final results.

The stresses in the concrete and the steel shell were calculated on the basis of the combined effects of the axial force and the bending moment. The limiting values for the stresses were the same as discussed in reference to the composite piles. Where necessary, the pipe was permitted to take some tension and this was considered safe in view of its 3/8" wall thickness. The maximum soil pressure available at the top of the sand stratum was assumed as one-half of the difference between the passive and active pressures in the sand. In only one case did this limiting value govern the load on the pile and in most cases only a small fraction of the available restraining force was developed.

In the instance of Pile No. 15-S2 shown in Figure 13, by installing a strut at the top of the pile capable of providing a horizontal force of 4.7 kips it was found that the full design load of 60 kips could be applied. Under the influence of this load, a restraining force of 1.15 kips only was required, at the top of the sand bed 4.25 kips per foot being available.

Number of Case B Piles Analyzed. Of the 24 Case B piles measured in the field and analyzed, 4 were found to be less than 2% out of plumb and thus could be immediately assigned a design load of 30 tons without analyses, 11 when analyzed were found to be able to safely carry



FORCES ACTING ON
CASE B PILE

(STEEL PIPE PILE)

FIGURE 13

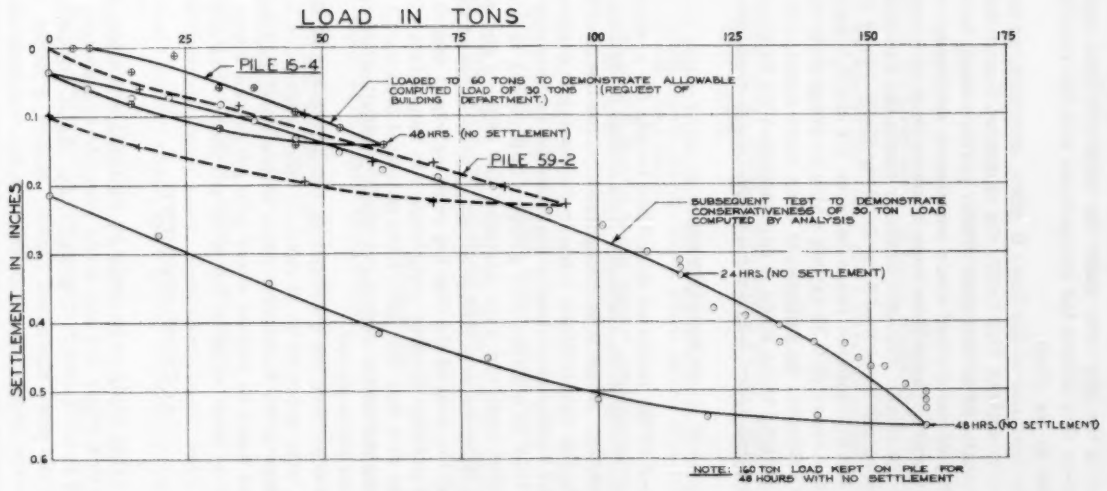
30 tons, 8 were assigned loads varying from 75% to 100% of the design load and one had to be assigned a design load of 17 tons.

Pile Load Tests

In order to demonstrate the performance of the Case A dog-leg composite piles analyzed, the New York City Department of Housing and Buildings required that a minimum of two of these piles be load tested. The tests were requested to be performed in accordance with the requirements stipulated in the New York City Building Code and thus the piles had to be subjected to twice the allowable design load computed on the basis of the analyses. Accordingly, representative piles of each of the two types discussed above under Case A were selected. Each, however, exhibited inclination in excess of the average and had been assigned a design load based on the results of analysis somewhat less than the allowable load of $57\frac{1}{2}$ tons permitted on a straight pile. Pile No. 59-2, representative of piles whose pipe sections were found to be tangent to the shell sections was assigned a design load of 47 tons and Pile No. 15-4 whose pipe section was not tangent to the shell section was permitted a load of only 30 tons. The load settlement diagram for each of these piles are included in Figure 14.

As may be seen in Figure 14 the total settlement in Pile No. 59-2 under a load of 94 tons was 0.227 inches and upon the removal of the load it rebound to 0.1 inches. Pile No. 15-4 under the influence of 60 tons showed a total settlement of only 0.142 inches and rebounded to 0.0034 inches upon the removal of load. As required by the Code, in the instance of each of these tests twice the design load was required to be maintained on the pile until a period of not less than 48 hours had passed without any observable settlement having occurred. As may be judged from the order of magnitude of the settlements observed, they were substantially below the maximum permitted by the Code and thus satisfactorily fulfilled the requirements of the city in this connection. Actually the maximum settlements experienced by these piles were less than the average observed for comparable loads on the straight piles. This, we believe, may be accountable to the fact that the time interval between driving and load testing was somewhat greater in the instance of the dog-leg piles.

For the purpose of demonstrating the conservativeness of the assumptions made in connection with the analyses for the determination of the design loads for the dog-leg piles, one of the two piles above described, namely, Pile No. 15-4, was subjected to a second load test and this time loaded to 160 tons which is about 5.3 times its design load. The results are included in Figure 14 and as may be seen the gross settlement experience under the 160 ton load was only 0.55 inches. The maximum load in this instance was maintained on the pile for a period of 48 hours without experiencing settlement before being removed. This test has also been plotted in Figure 2 for comparison with the results on straight piles. The results of the load test, we believe demonstrate conclusively that the method of analysis adopted render loads which are without question safe and on the conservative side. The degree of conservativeness,



LOAD TESTS ON DOG-LEG PILES
No's 15-4 & 59-2
CASE A

FIGURE 14

however, cannot be evaluated on the basis of the load test results alone since the stresses in the pile even under the maximum load applied cannot be determined by this means for comparison with the limiting stresses permitted by the Code.

The City, in the instance of the Case B piles, accepted, on the basis of the analyses alone without load tests, the computed design loads for the dog-leg piles analyzed at this site. Their performance, however, has been thoroughly demonstrated since continuous settlement observations, on a semi-monthly basis have been made on each of the columns supported on dog-leg piles. The observations cover the entire period of building construction and as of this date which is about a year after their installation the settlements range between 0.1 and 0.5 inches, the average being of the order of 0.25 inches which is about their elastic shortening. This evidence, we believe, is even more conclusive than that provided by the load tests since it demonstrates the favorable performance of such piles under load over a much longer time period.

CONCLUSIONS

The development of the facilities described herein for analyzing dog-leg piles afford an inexpensive, rapid and safe means of evaluating their load carrying capacity. While it is recognized, on the basis of the load test presented that the assumptions made in the analyses employed in the two cases cited rendered allowable loads on the piles which were on the conservative side, of considerably greater importance is the fact that the dog-leg piles analyzed were salvaged and utilized and not abandoned as has long been the accepted practice. In the instance of the Case A piles alone the cost of the dog-leg piles salvaged was in excess of $30,000, and of perhaps greater significance in connection with the Case B piles is the fact that the piles here salvaged eliminated the necessity of removing the entire existing building foundation and mucking out the obstructions encountered.

The procedure described for determining the safe loads on dog-leg piles, we believe, may prove to be of value to other engineers in effecting considerable savings in the foundation costs where steel encased or pipe piles are employed since on almost every pile job driven where long piles are required at least some of the piles are found to be dog-legged.

ACKNOWLEDGMENT

Suggestions for this study were made by Philip C. Rutledge, M. ASCE who has reviewed this paper and has made valuable suggestions which were incorporated in it. Louis J. Capozzoli, Jr., ASCE carried out the actual analyses of the piles discussed herein. Moran, Proctor, Mueser and Rutledge were the foundation consultants on the two projects referred to where dog-leg piles were analyzed.

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a. Presented at the New York (N.Y.) Convention of the Society in October, 1953.
b. Beginning with "Proceedings-Separate No. 290," published in October, 1953, an automatic distribution of papers was inaugurated, as outlined in "Civil Engineering," June, 1953, page 66.
c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
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